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DISCUSSION

Proc. Paper 16006

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INFORMATION RETRIEVAL

The key words, abstract, and reference "cards" for each article in this Journal represent part of the ASCE participation in the EJC information retrieval plan. The retrieval data are placed herein so that each can be cut out, placed on a 3 × 5 card and given an accession number for the user's file. The accession number is then entered on key word cards so that the user can subsequently match key words to choose the articles he wishes. Details of this program were given in an August, 1962 article in CIVIL ENGINEERING, reprints of which are available on request to ASCE headquarters.

*Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

16032 GEOMETRY OF TIDAL INLETS: EQUATIONS

KEY WORDS: Channels; **Empirical equations; Inlets (waterways);** Sediment transport; **Storm surges; Tides;** Waterways

ABSTRACT: The geometry of tidal inlet ebb shoal, main channel and cross section is investigated. Formulae interrelating depths, length, and area parameters show a high degree of coherence in inlet geometry, with the cross-sectional area of the inlet a fundamental parameter.

REFERENCE: Vincent, Charles L., and Corson, William D., "Geometry of Tidal Inlets: Empirical Equations," *Journal of the Waterway, Port, Coastal and Ocean Division,* ASCE, Vol. 107, No. WW1, **Proc. Paper 16032**, February, 1981, pp. 1-9

16049 COASTAL ENGINEERING AND CONSTRUCTION IN JAPAN

KEY WORDS: **Barges;** Breakwaters; **Bridges;** Caissons; **Coastal engineering; Construction;** Harbors; **Japan;** Offshore structures; Piers; Ports; **Underwater construction**

ABSTRACT: The selected findings of the four Americans who participated in ASCE's recent U.S.-Japan exchange of eminent engineers with specialties in underwater structures are outlined. Particularly in coastal engineering, Americans can learn from the Japanese. Like the United States, Japan is an economically and technologically complex advanced industrial nation, and it too is seeking to accommodate social and environmental considerations in its engineering and construction planning. Three projects are described to illustrate the coastal engineering and construction works studied by the American visitors in Japan. They are: the recently completed Kahima port-industrial area, the Mitsubishi floating petroleum storage system (now being designed for a site in the Goto Islands west of Kyushu), and the Honshu-Shikoku Bridge Project, which is now under construction.

REFERENCE: Paulson, Boyd C., Jr., "Coastal Engineering and Construction in Japan," *Journal of the Waterway, Port, Coastal and Ocean Division,* ASCE, Vol. 107, No. WW1, **Proc. Paper 16049**, February, 1981, pp. 11-26

U.S. Customary-SI Conversion Factors

In accordance with the October, 1970 action of the ASCE Board of Direction, which stated that all publications of the Society should list all measurements in both U.S. Customary and SI (International System) units, the following list contains conversion factors to enable readers to compute the SI unit values of measurements. A complete guide to the SI system and its use has been published by the American Society for Testing and Materials. Copies of this publication (ASTM E-380) can be purchased from ASCE at a price of $3.00 each; orders must be prepaid.

All authors of *Journal* papers are being asked to prepare their papers in this dual-unit format. To provide preliminary assistance to authors, the following list of conversion factors and guides are recommended by the ASCE Committee on Metrication.

To convert	To	Multiply by
inches (in.)	millimeters (mm)	25.4
feet (ft)	meters (m)	0.305
yards (yd)	meters (m)	0.914
miles (miles)	kilometers (km)	1.61
square inches (sq in.)	square millimeters (mm^2)	645
square feet (sq ft)	square meters (m^2)	0.093
square yards (sq yd)	square meters (m^2)	0.836
square miles (sq miles)	square kilometers (km^2)	2.59
acres (acre)	hectares (ha)	0.405
cubic inches (cu in.)	cubic millimeters (mm^3)	16,400
cubic feet (cu ft)	cubic meters (m^3)	0.028
cubic yards (cu yd)	cubic meters (m^3)	0.765
pounds (lb) mass	kilograms (kg)	0.453
tons (ton) mass	kilograms (kg)	907
pound force (lbf)	newtons (N)	4.45
kilogram force (kgf)	newtons (N)	9.81
pounds per square foot (psf)	pascals (Pa)	47.9
pounds per square inch (psi)	kilopascals (kPa)	6.89
U.S. gallons (gal)	liters (L)	3.79
acre-feet (acre-ft)	cubic meters (m^3)	1,233

JOURNAL OF
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Geometry of Tidal Inlets: Empirical Equations

By Charles L. Vincent,[1] M. ASCE and William D. Corson[2]

Introduction

The engineering of tidal entrances is made difficult because the relative influence of short period wind waves and the tide varies with location and time in an inlet system with the hydrodynamics often a mixture of both. Although progress has been made in the numerical modeling of tides and storm surges in bays, no widely accepted model exists that couples short and long period waves with sediment transport equations. The coastal engineer must combine empirical methods or experience with hydraulic data from numerical or physical models to predict inlet response to dredging, structural modification, or even to natural changes. Although there is information on the relationship between tidal prism and cross-sectional area (References 1 and 2), there are no widely accepted methods for estimating changes in the physiographic parameters of an inlet such as channel lengths and depths. This paper presents data that demonstrate the existence of a few simple empirical relationships among such parameters.

The results to be presented are relationships only between physiographic parameters and do not consider wave and tide variables although such research is desirable. It was assumed that were an inlet in equilibrium with its hydraulic environment, inlet geometry would adjust to a stable form. Examination of physiographic parameters alone would show any characteristic variations in inlet geometry and information on whether inlet geometry tends toward an equilibrium.

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Note.—Discussion open until July 1, 1981. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on December 11, 1979. This paper is part of the Journal of the Waterway, Port, Coastal and Ocean Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. WW1, February, 1981.

Further, the choice and measurement of wave and tide parameters is not clear nor are the appropriate data widely available.

DATA

Sixty-seven inlets were selected for study. The inlets are on the Atlantic and Pacific coasts and on the Gulf of Mexico; a few are jettied and some may have been dredged. The charts used were National Ocean Survey boat sheets and the dates varied considerably. This sampling scheme was chosen

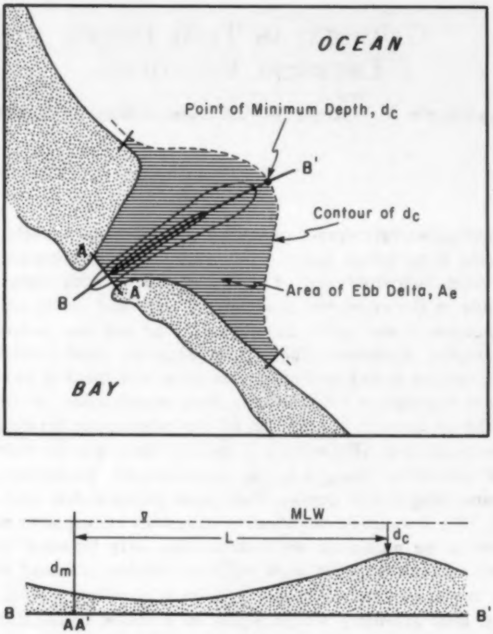

FIG. 1.—Definition of Parameters d_a, d_c, d_m, W, L, A_E: Width, W, is Length AA' and Average Depth of Cross Section along AA' is d_a

to assure that a variety of conditions were analyzed. The presence of structures and possible dredging are believed only to add random scatter in any relationship.

The parameters (Fig. 1) measured to quantify inlet physiography were the width, W, average and maximum depths, d_a, and d_m in the cross section of the inlet throat at minimum inlet width; the length, L, of channel from the minimum width line to the shallowest depth, d_c, in the channel as it passes across the edge of the ebb shoal. Finally, a measurement of area, A_E, for the ebb delta data was estimated by contouring the previously defined depth

d_c, and calculating the area bounded by this contour, the minimum inlet width line and orthogonals on either side of inlet where the contour of d_c approach a direction parallel with the coast.

From integration of a sample of inlet minimum width cross sections, it was found that the area of this cross section, A_c, was very close to the product

$$A_c = W \times d_a \quad (1)$$

and A_c was so estimated for this paper. It should be noted that the cross-sectional area at minimum width is not equal to the area of the minimum cross section, but the minimum width line is much easier to find.

Empirical Formulae

The parameters were plotted against each other and those plots that showed promise were analyzed through standard linear and power curve fitting analysis. Either a linear analysis or a power curve analysis was performed. For a relationship

TABLE 1.—Regression Results

Independent variables (1)	Dependent variables (2)	Curve fitted (3)	R^2 (4)	F-ratio (5)
A_c	A_E	power	80.0	260
A_c	L	power	77.5	238
A_c	d_m	power	67.6	133
A_c	d_c	power	50.2	66
L	A_E	power	83.4	327
d_m	d_a	linear	76.8	215
d_m	d_c	power	57.9	89
d_m	L	linear	48.0	60
W/L	da/dc	power	32.1	31

to be accepted with a 5% level of significance on the basis of 67 inlets, the analysis must produce an F-ratio value greater than 7.08 with 1° and 65° of freedom. Table 1 provides the F-ratio and the coefficient of determination, R^2, for the relationships prescribed in this paper. The larger the F-ratio and R^2 value, the better the curve is presumed to fit the data.

The most significant parameter found was A_c. The formulae for A_E, L, d_m, and d_c are given below with A_c in units of feet squared:

$$A_E = 3.148 \times 10^{-5} A_c^{1.04} \quad (2)$$

in which A_E is measured in square miles

$$L = 23.92 A_c^{0.55} \quad (3)$$

in which L is measured in feet

$$d_m = 0.5479 A_c^{0.38} \quad (4)$$

in which d_m is measured in feet and

$$d_c = 0.2367 A_c^{0.34} \qquad (5)$$

in which d_c is measured in feet. Fig. 2 provides the scatter plots that go with Eqs. 2 through 5. On this figure, inlets are categorized by Atlantic Ocean, Pacific Ocean, and Gulf of Mexico. No major geographic trend seems evident.

In order to understand why Eqs. 2 and 3 indicate strong relationships (R^2 of 80% and 77.5% respectively), it is important to consider the significance of the parameter d_c on which the definition of A_E and L is dependent. Parameter

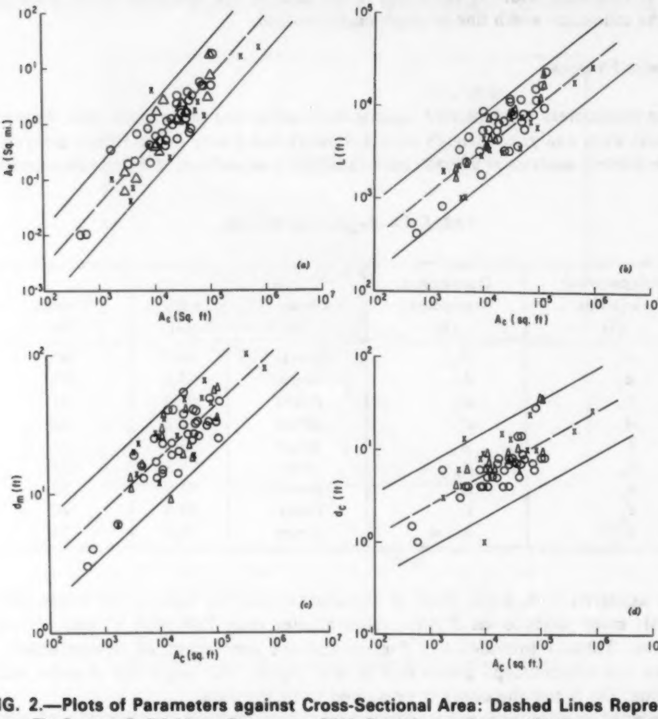

FIG. 2.—Plots of Parameters against Cross-Sectional Area: Dashed Lines Represent Best Fit Curve; Solid Lines Represent 95% Confidence Bands (Circles are Atlantic Inlets, Triangles are Gulf of Mexico Inlets, and x's are Pacific Inlets)

d_c was defined as that depth in the main channel which was shallowest as the channel crossed the ebb shoal. This depth may characterize that location in the channel where wave-induced sediment transport and ebb tide induced sediment transport are balanced in the average. The parameter L defined as the distance along the channel from the minimum inlet width line to the point with depth d_c can be considered a physiographic parameter characteristic of wave-tide regime of an inlet. Likewise, the use of d_c to define an ebb delta area appears to define an area characteristic of the inlet influence. Walton

(Ref. 3) has shown a dependence of volume of the ebb delta as a function of cross-sectioned area. The cross-sectional area of the gorge as related to the tidal regime of inlet has been recognized as important (Ref. 2). Although A_c here is not defined as the minimum cross section as in Ref. 1, the values should be correlated.

The relationship of A_c and d_m has a high R^2 value of 67.6%. The relationship of d_c to A_c is with an R^2 value of 50.2%. That d_c and A_c are not more highly related is not unexpected, because d_c should be more strongly reflective of

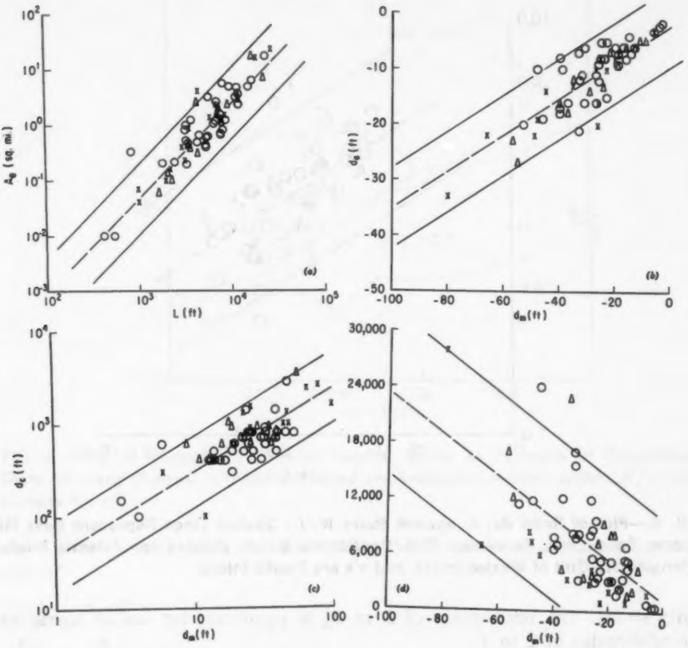

FIG. 3.—Cross Plots of Selected Parameters: Dashed Lines Represent Best Fit Curve; Solid Lines Represent 95% Confidence Bands (Circles are Atlantic Inlets, Triangles are Gulf of Mexico Inlets, and x's are Pacific Inlets)

the wave regime of the inlet while A_c may be more dominated by the tide. Parameters W and d_a were not plotted against A_c because of their use in Eq. 1 to define A_c.

The following relationships were also found:

$$A_E = 3.9245 \times 10^{-7} L^{1.71} \quad (6)$$

$$d_a = 1.42 - 0.347\, d_m \quad (7)$$

$$|d_c| = 0.5662\, |dm|^{0.78} \quad (8)$$

$$L = 539 - 226.7\, d_m \qquad (9)$$

In Eqs. 6 through 9, d_a, d_c, and d_m were negatively defined, i.e., for an average depth of 10 ft, $d_a = -10$. Fig. 3 provides the accompanying scatter-grams.

The strong relationship (R^2 of 83.1%) for Eq. 6 is somewhat unexpected. Although it can be argued that the ebb delta is semicircular, and, therefore, that A_E should be proportional to L^2, examination of L and ebb delta shape indicated a diversity of shapes with the channel often not following the shortest course to the ebb delta edge. The relationship of d_a to d_m, and d_c to d_m are

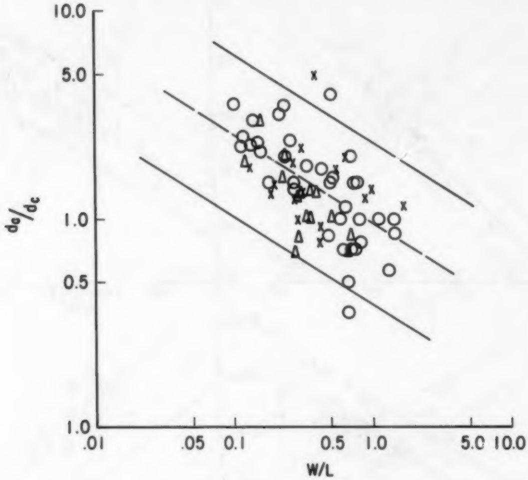

FIG. 4.—Plot of Ratio *da/dc* against Ratio *W/L*: Dashed Lines Represent Best Fit Curve; Solid Lines Represent 95% Confidence Bands (Circles are Atlantic Inlets, Triangles are Gulf of Mexico Inlets, and x's are Pacific Inlets)

fairly strong. The relationship of L to d_m is significant but not as strong as the relationship of L to A_c.

Other Results

Two other results dependent upon the ratio of inlet width to length (W/L) were found. The first is a relationship between the ratio of d_a to d_c to W/L given by

$$\frac{d_a}{d_c} = 0.9289 \left(\frac{W}{L}\right)^{-0.42} \qquad (10)$$

It has as an F value of 30.8 and an R^2 value of 32.1%. The relationship is significant well above a 5% level but there is appreciable scatter. The relationship indicates that for inlets narrow with respect to their channel length, the throat

depth tends to be much deeper than the critical depth on the ebb delta (Fig. 4). For W/L of about .1, d_a is about 1.5–2 times d_c, however, as W/L increases, d_a/d_c decreases until the critical depth on the ebb delta may be greater than the average depth in the inlet throat.

Eq. 10 motivated a nondimensionalization of the channel profile. The depth at point $X = \alpha L$, $0 < \alpha < 1$ along a channel was divided by d_m, the deepest

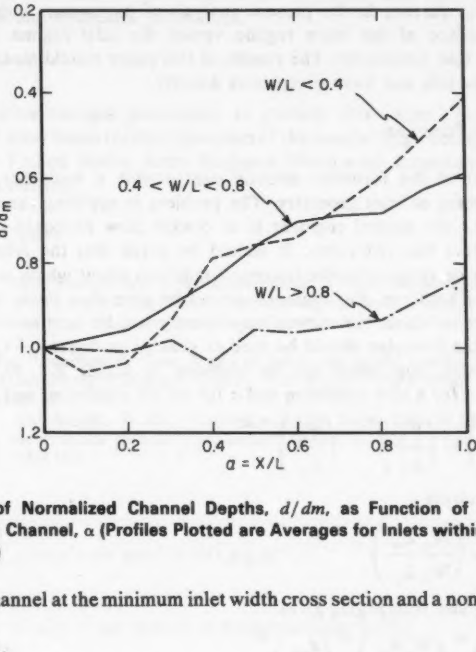

FIG. 5.—Plot of Normalized Channel Depths, d/dm, as Function of Proportional Distance along Channel, α (Profiles Plotted are Averages for Inlets within W/L Ratio Ranges Noted)

depth in the channel at the minimum inlet width cross section and a nondimensional plot of

$$\frac{d(x)}{d_m} = \frac{d(\alpha L)}{d_m} \quad \text{(11)}$$

was plotted as a function of a dimensionless distance, α. For different ranges of W/L, the profiles were averaged and plotted in Fig. 5. The average channel profile for inlets with W/L small has a distinctly higher bar than the channels or inlets with W/L large.

Analysis

The nine relationships presented suggest considerable coherence of inlet physiography. That cross-sectional area is a major parameter useful in predicting other elements of inlet geometry is not surprising given the tidal-prism cross-sectional area relationship of O'Brien (Refs. 1 and 2). The flow of water in and out of an inlet keeps the inlet open, with the tidal prism a measure of this

flow volume. O'Brien's relationship indicates that there is a strong relationship between the flow and the size of the inlet cross section which is a major element of inlet geometry. Strong relationships between the cross-sectional area and other physiographic elements imply that the tidal dominance has an equally important role in controlling the physiography of the ebb delta and main channel.

Although the curve-fit analyses indicate highly significant correlations among the parameters, it is important to consider the unexplained variance. This variance represents the absence in the present analysis of parameters indicative of the relative influence of the wave regime versus the tidal regime or direct use of wave and tide parameters. The results of this paper should motivate research to incorporate tide and wave parameters directly.

Application of Formulae

Application of the formulae directly can provide a basis for estimates of the readjustment of inlet geometry. The problem in applying the formulae that still confronts the coastal engineer is to predict how proposed change at the inlet will affect the hydraulics. It should be noted that the relationships are based on a wide range of hydrodynamic conditions about which no information was known. Therefore, the engineer cannot be sure that those factors which introduce scatter about the curves may or may not be dominant in a specific case. Thus, the formulae should be used as general guidelines.

An interesting final result can be obtained by taking Eq. 10, subscripting variables by n for a new condition and o for an old condition, and then dividing

$$\left(\frac{d_{an}}{d_{cn}}\right)\left(\frac{d_{ao}}{d_{co}}\right)=\left(\frac{W_n}{L_n}\right)^{-0.42}\left(\frac{W_o}{L_o}\right)^{-0.42} \quad \text{(12)}$$

Rearranging yields

$$\left(\frac{d_{an}}{d_{cn}}\frac{d_{co}}{d_{ao}}\right)=\left(\frac{W_n}{W_o}\frac{L_o}{L_n}\right)^{-0.42} \quad \text{(13)}$$

and inverting and rearranging gives

$$\frac{d_{cn}}{d_{co}}=\left(\frac{L_o}{L_n}\right)^{.42}\left(\frac{W_n d_{an}}{W_o d_{ao}}\right)^{.42}\left(\frac{d_{an}}{d_{ao}}\right)^{.58} \quad \text{(14)}$$

If in modifying an inlet, the cross section remains roughly constant, Eq. 13 becomes

$$\frac{d_{cn}}{d_{co}}=\left(\frac{L_o}{L_n}\right)^{.42}\left(\frac{d_{an}}{d_{ao}}\right)^{.58} \quad \text{(15)}$$

If the channel length does not change much, Eq. 15 indicates that the ratio of the new depth of the shallow point in the channel on the outer bar to the old will only change as the 0.58 power of the ratio of new average depth in the minimum cross section to the old. This implies that only a major change in the depth of the cross section will lead to a significant change in the depth d_c. The formula, however, does not contain any effects jetties would have in blocking sediment transport into the channel or in concentrating the flow.

Summary

Ten relationships descriptive of tidal inlet physiography have been presented. The data suggest a fairly strong coherence of inlet geometry from which it is inferred that inlet geometry tends to be self adjusting to wave-tide regime. The results are sufficiently encouraging to suggest their use in the planning of inlet modifications and to motivate further research into the relationship of physiographic and hydraulic parameters.

Acknowledgments

The writer acknowledges permission to publish this paper. The General Investigation of Tidal Inlets research program of the coastal Engineering Research Center and the United States Army Engineer Waterways Experiment Station sponsored this research.

Appendix I.—References

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Appendix II.—Notation

The following symbols are used in this paper:

A_E = area of ebb shoal;
A_c = area of inlet cross section at minimum inlet width;
d = depth;
d_a = average depth of cross section at minimum inlet width;
d_c = shallowest depth in channel;
d_m = maximum depth in cross section at minimum inlet widths;
L = channel length;
X = distance along channel;
W = minimum inlet width; and
α = proportional length.

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Coastal Engineering and Construction in Japan

By Boyd C. Paulson, Jr.,[1] M. ASCE

Although Japan's land area of 147,200 sq miles (370,000 sq km) is smaller than that of California's 159,000 sq miles (411,000 sq km), its 16,470-miles (26,500-km) coastline is longer than that of the continental United States (13,000 miles or 21,000 km), and provides many good harbors. With four main islands and 3,000 smaller ones spread out along a 2,800-miles (4,500-km) archipelago, it has historically depended upon seaways for its primary transportation. Like England, this island nation of 115,000,000 people became a maritime trading country. In the last few decades it has thus developed the second-largest economy in the world. It is natural, then, that the Japanese have given considerable attention to the engineering and construction of coastal structures, and in technology and procedures they have a great deal to offer the rest of the world. They also have a strong interest in what is happening in other countries, particularly in deeper offshore structures as they move farther from land in search for the food and mineral resources to sustain their economy.

United States-Japan Exchange

As a mechanism to foster the exchange of information in this vital field, the United States National Science Foundation (NSF) provided a grant to the ASCE to support the American side of an exchange of eminent civil engineers with specialties in underwater structures (report by Robert Morgan, 1979). Simultaneously, the Japan Society for the Promotion of Science (JSPS) funded the Japanese expenses for the exchange.

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Note.—Discussion open until July 1, 1981. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on December 19, 1979. This paper is part of the Journal of the Waterway, Port, Coastal and Ocean Division, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. WW1, February, 1981. ISSN 0148-9895/81/0001-0011/$01.00.

The ASCE sought the help of its Coastal Engineering Research Council, which in turn established a Project Steering Committee to provide technical guidance and monitoring for the project in the United States. A similar mechanism was set up in Japan. Each country's committee developed a large list of its nation's experts in underwater structures, and exchanged these lists. The Japanese then designated their choices of the Americans they would like to have visit Japan, and the Americans selected the Japanese whom they thought would be of greatest interest to Americans. The seven eminent engineers who finally participated in this exchange are listed in Appendix I. The Americans whose findings are surveyed here are Arthur R. Anderson, Ben C. Gerwick, Jr., John H. Nath, and John R. Paulling.

Each of the participants spent from a few weeks to three months in the host country. Their activities mainly consisted of two kinds: (1) Disseminating information in lectures and meetings to share knowledge about research and practice in their home countries; and (2) acquiring information in the host countries by visiting research laboratories, projects, private and governmental engineering and construction offices, and by attending meetings and conferences.

The purpose of this paper is to highlight some of the findings of the four Americans who went to Japan. It is based upon interviews with each of these four, upon the reports and papers they prepared (1–4,7,8,10), and upon written and graphical materials they brought back from Japan. The writer himself spent six months in Japan in 1978 on a study of the Japanese construction industry funded by another NSF project (11) and spent an additional month in 1979. While in Japan he visited some of the same sites as the four ASCE visitors, and will also draw upon this background to support the findings of the ASCE visitors.

Impressions

At present Japan's strength in the field of underwater structures appears to be more in coastal and harbor works than in offshore ocean structures. The latter are more highly developed in the United States, largely because of America's more extensive involvement with the exploration and production of petroleum resources, and because of the strong interest of the United States Navy in some kinds of offshore works. Japan is now giving stronger emphasis to ocean structures, however, largely as a way of redirecting the partially idled resources of its advanced, high-capacity shipbuilding industry, so an expanded level of offshore activity can be expected in the future. But even now Americans can learn a great deal from Japan's engineering and construction of coastal structures.

Ambitious works currently planned or under construction in Japan include the world's longest bridge span; the world's longest tunnel which is being driven under the sea to connect the islands of Honshu and Hokkaido; underwater double-hulled concrete vessels for storage of oil and liquified natural gas (LNG); two floating airports; several underwater tube-tunnels; concrete tanks for offshore fish farms; and a number of industrial harbor developments on a grand scale similar to those in Long Beach, California and Jubail, Saudi Arabia. These projects are not isolated curiosities intended primarily to set world records. Rather, they are an important part of Japan's national development plans, and

are being undertaken by a massive, technologically advanced engineering-construction industry that accounts for over 20% of the gross national product in the world's second-largest economy. Three of these projects will be described in later sections of this paper.

Japan's engineering and construction projects and its rapid technological and economic advancement are supported by construction research and development efforts that are unparalleled in this industry. These efforts are not confined to government agencies, universities, and manufacturers. Almost all major construction contractors maintain large, internally funded research laboratories that are an integral part of the construction business in Japan. The laboratories in all of these organizations include some of the largest and best-equipped facilities in the world. Owing to its importance to Japanese engineering and construction, research and development will be examined in more detail in a later section.

Other general impressions about coastal engineering and construction in Japan relate to its response to environmental and social constraints. It is well known that with so many people living in such a small space, the impact of development on the environment has at times courted disaster. What is less well known, however, is that the Japanese government, with a surprising degree of cooperation from industry, has imposed some of the world's strictest regulations to preserve or restore the quality of air and water, and to prevent noise, vibration, soil contamination, land settlement, and odors. Crowding and earlier displacements of people have bred increasing social and political opposition to new construction and land acquisition; for their very survival engineers and contractors must be good neighbors, and every square meter made available for development must be used to maximum benefit for the community as well as for the project at hand (11).

Research and Development

The real beginnings of modern construction-and-engineering research and development patterns in Japan came shortly after the end of World War II. For the construction industry there was a massive rebuilding job ahead, and it desperately needed to advance its technology and procedures to incorporate the best of what was available in the world. In the early post-war years a few of the largest construction companies established research facilities, and subsequently most of the larger companies followed this pattern (11). There was also a major expansion of research and development facilities and efforts in government public works agencies and in universities. Initially, the main purposes were studying Japan's reconstruction needs, and analyzing technologies, domestic and foreign, that could most beneficially be assimilated and applied to the reconstruction effort.

Research Facilities.—Major research facilities among over 15 laboratories visited by the Americans in Japan included those devoted primarily to naval architecture and offshore structural research, those associated with port and harbor structures, dedicated facilities for particular local needs, and general engineering and construction laboratories in private industry.

The naval architecture facilities not only study ship design and fabrication, but also are doing research on offshore structures such as oil-drilling platforms. The Ministry of Transportation's Ship Research Institute at Mitaka, near Tokyo,

is conducting research on wave-induced motions and loads, and structural response and mooring of ocean structures. Its principal facility is a new wave-current tank 28 m × 40 m × 2 m deep (92 ft × 131 ft × 7 ft) (10). Two projects underway were model tests for the floating structure proposed to support the new Osaka International Airport and for the floating offshore petroleum facilities to be built in Kyushu. Two important university laboratories include the University of Tokyo's combined seakeeping/maneuvering test tank located in Chiba City and Kyushu University, both of which are studying hydrodynamic and structural performance of offshore structures. Finally, shipbuilding companies such as Mitsui and Mitsubishi maintain extensive facilities nearby their various large shipyards. In contrast to United States practice, where design is by a consulting firm separate from the builder and where model-testing might be carried out in an independent commercial wave tank, Japanese shipbuilders integrate design, testing, and fabrication in the same company and usually at the same site. This is important because the design of ships and offshore structures still depends greatly on empirical testing; as they move more and more into offshore structures, the Japanese may have a definite advantage here. Furthermore, since Americans are largely dependent upon overseas facilities for testing, the host countries can closely observe the results of any design advances we might develop. Testing in the Japanese shipyard laboratories includes models of triangular tension-leg platforms, the Aquapolis floating city for the 1975 Okinawa Exposition, structural joints for offshore platforms, semisubmersible drilling platforms, jack-ups, construction barges, a 5,000-ton-capacity twin-hulled derrick barge, and LNG storage tanks (10).

The Ministry of Transportation's Port and Harbor Research Center is world-renowned for research in its field. It employs approx 200 people in research and 60 in administration (8). At the time of the 1978 visits, a large hydraulic model of Osaka Bay was being used for studies of tidal movements and the effects of proposed new construction projects, including the offshore airport (10). Its horizontal scale was 1:2,000, the vertical scale was 1:160, and it was using a 4-min tidal cycle (8). Other studies included wind wave generation, wave pressures on breakwaters, and behavior of moored ships (10). In addition, many studies of sediment transport along the coasts were in progress, and wave-absorbing structures to damp and minimize refraction of waves in harbors were being tested.

Local facilities related to coastal engineering include the Sapporo Experimental Station of the Hokkaido Bureau of Development, and the wave basin laboratory near Niigata. Among other activities, the Hokkaido facility is conducting research on "designs to absorb and dampen wave energy impinging on coastal and offshore breakwaters." The tests are being correlated with field tests on the large industrial port development now under construction at East Tomakomai, as well as other smaller installations on the coasts of Hokkaido (2). The Niigata facility was set up to help study major problems caused by construction developments, and the tendency of earthquakes to liquify the sands that then sink some of the thousands of tetrapods that protect the coastline. Niigata's 50 m × 50 m (164 ft × 164 ft) wave basin is used to study wave action on existing and proposed future port geometry (8).

Private industry research facilities visited by American engineers included the Central Research Institute of Electric Power Industries (CRIEP) and the

engineering-and-construction laboratories of Kajima Corporation and Ohbayashi-Gumi, both of which are top-5 contractors. CRIEP receives its base funding from 0.2% of the gross receipts from the sale of power by the nine member utilities [a mechanism similar to the funding for the Electric Power Research Institute (EPRI) in the United States], and also receives project funding over and above this for specific projects of interest to a particular funding organization. The Institute gets involved in coastal engineering because of the need for cooling water, outfalls, etc. for thermal and nuclear power plants. The laboratory includes many wave flumes and basins, and does considerable research on coastal problems, including onshore-offshore sand transport in waves (8).

FIG. 1.—Ohbayashi-Gumi, Ltd., Technical Research Institute, Located in Kiyose, Tokyo, Japan

FIG. 2.—Hydraulic Wave Basin at Kajima Corporation Technical Research Institute (58 m long × 20 m wide × 1.5 m deep)

An aerial view of the Ohbayashi-Gumi Technical Research Institute is shown in Fig. 1. A large wave basin for the Kajima Corporation Institute of Construction Technology is shown in Fig. 2. Like those of most large contractors, these laboratories conduct diversified research ranging through all areas of construction, including coastal engineering and offshore works. Kajima, for example, "is actively engaged in improving coastal public works from the viewpoint of increasing safety and reducing costs and deleterious effects to the environment. . . . Efficient breakwaters and absorbing walls for low-wave reflection are being investigated as well" (8). About 10% of the research in these laboratories is done for outside clients such as oil and gas companies or public agencies,

but the bulk of their work is internally funded.

Visitors' Opinions on Japanese Research.—The American engineers visiting Japan were uniformly impressed with the large and sophisticated research facilities available for coastal engineering and construction. By and large, they also spoke favorably of the positive institutional attitudes and funding that supports research and development in government agencies, universities, and private corporations.

In the realm of conceptual and basic research, the visitors seemed to think that the ground is at least as fertile for innovation in the United States, particularly among young researchers whose individual accomplishments are possibly more readily recognized. In Japan such achievements are more likely to be attributed to the institution as a whole, or sometimes to senior researchers who have high administrative responsibilities, much like the traditional European system. It is significant that many of the advanced computer and analytical techniques used in Japan (NASTRAN, SAP, etc.) are of American origin, as are some of the design concepts which are seeing their first large-scale laboratory testing and field implementation in Japan. Nevertheless, with excellent laboratory facilities and a more favorable institutional climate for implementation of new concepts, Japan has a definite advantage in the applied research and developmental phases of getting new concepts into practice.

One possible drawback to the ready implementation, according to two of the visitors, is that some applications find their way to expensive full-scale implementation before there has been enough applied research and development, and a sufficiently thorough technical and economic evaluation of alternative systems (2,8). For example, in one major breakwater project several concepts are being implemented simultaneously even though it seems that some ought to have been eliminated earlier. Furthermore, given that the concurrent implementation is going ahead, it would seem that there could be more consideration given to instrumentation and monitoring to document which system performs best under the given site conditions.

As in the United States and elsewhere in the world, there is a tendency toward compartmentalization of research, which leads to some duplication of work carried out elsewhere, perhaps in other disciplines. As happens everywhere, the arrival of a visitor also opens up local communication channels among researchers who normally do not visit each other's laboratories and projects in their regular activities. Such compartmentalization also reinforces a natural but unproductive tendency to optimize individual operations and components rather than the overall system. For example, there are hundreds of thousands of tetrapods on one coast to control erosion and sediment transport along the beaches (problems largely caused by man-made works), and much of the sand and aggregate for the concrete in the tetrapods is dredged from the rivers which would otherwise naturally transport the deposits to the sea where they would replenish the materials being eroded. This, of course, greatly oversimplifies the problems in this case, but it illustrates the point. Again, such suboptimization is a world-wide phenomenon, and to the credit of the Japanese they are becoming more and more astute in recognizing and correcting these system-wide problems.

Finally, it is a common misconception that massive government subsidies enable the Japanese to carry out research and development in engineering and construction. Comparing countries at a national level for all industries, approximately two-thirds of research and development in the United States is government

funded, and much of that is directed towards defense applications, while in Japan about two-thirds of the research and development is funded by the private sector, and the vast majority of that is directed towards commercial and civilian applications (9).

The significance of this is that, given the proper economic and institutional climate, it is indeed possible for the private sector of the engineering and construction industry to support and benefit from a systematic research effort of the type one normally associates with capital-intensive manufacturing industries. Given that it is only a fairly recent development that has taken root and flourished in Japan, it is possible that the seeds for such institutions could be planted and successfully nurtured in other advanced industrialized countries. The incentives that drive Japanese research ought to operate just as well in the United States: creating new markets; solving practical field problems; corporate prestige; gaining a keener competitve edge; substantiating claims; and improving compliance with regulations. Perhaps by studying the evolution of construction research in Japan, a beginning can also be made in the United States (11).

FIG. 3.—Location of Three Major Projects in Japan

Projects

Three projects have been chosen here to illustrate the coastal engineering and construction works studied by the American visitors in Japan. They are the recently completed Kashima port-industrial area, the Mitsubishi floating petroleum storage system now being designed for a site in the Goto Islands west of Kyushu, and the Ohnaruto Bridge as part of the Honshu-Shikoku Bridge

Project which is now under construction. The locations of these projects are shown on Fig. 3.

Gerwick summed up the grandeur of these and other projects by commending "the political boldness of the Japanese government and people in pushing ahead with new port-industrial areas, new high-speed rail lines, new housing, and major bridge-tunnel connections within and between the major islands. These are of the same order and require the same national commitment as the U.S. Interstate Highway program of the 1960s" (2).

Kashima Port-Industrial Complex

This section is based upon Ref. 13 and interviews with two of the visitors to the site. Kashima is one of many industrial port developments in Japan. Between them the American engineers went to over half a dozen major port complexes, including Hakata, Tomakomai, Niigata, Kobe, Nagasaki, Hiroshima,

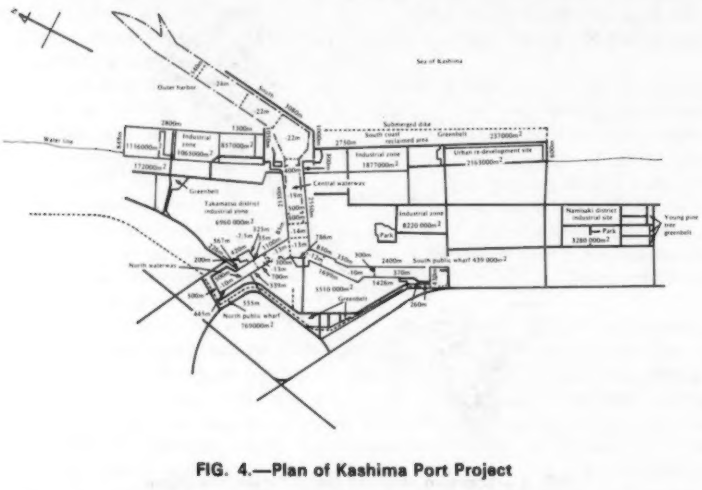

FIG. 4.—Plan of Kashima Port Project

Tamano, and Chiba as well as Kashima. Kashima is neither the largest nor the smallest, so perhaps it more typically illustrates the scale and ambition of engineering and economic planning in Japan.

Planning.—Kashima Port is located on Japan's Pacific Coastline, about 80 km (50 miles) northeast of Tokyo, and is the focal point of the newly developed Kashima Industrial Zone. Beginning with barren land, the first phase of the project began in 1963 and essentially was completed in 1975. The site was chosen for its proximity to Tokyo, for geological conditions that would provide good foundations for the various structures of the heavy chemical industry, and for its abundant supply of fresh water. Funding of approx $700,000,000 was shared by the Federal government (breakwaters and land reshaping) and private industry (dredging and pier construction) (8).

From the beginning, planners sought to reverse the decline of this hitherto economically backward area, yet somehow preserve the values and traditions of the farmers and others living there. Farmers were guaranteed compensating land plots near the site that were 50% larger than the poor land acquired for industrial development, and the government improved the new soil for higher farming productivity. The project was also designed to prevent air, water, and noise pollution.

Overview.—The Kashima Industrial Zone comprises an area of 750 sq km (290 sq miles), and extends 60 km (37 miles) along the coast and 15 km (9 miles) inland. The area surrounding the man-made harbor is shown on Fig. 4.

The breakwater extended north, angling about 30° from the shore, was designed in view of prevailing NNE winds, storm-driven waves tending eastward, and sand drift being prevalently from south to north. The outer channel and anchorage are designed to handle tankers of up to 200,000 dwt in a water depth of 22 m–24 m (72 ft–79 ft). The inner harbor has a channel width of 600 m (1,970 ft), a depth of 19 m (62 ft), and will accommodate carriers of up to 150,000 dwt. The public wharves at the north and south extremities of the inner harbor

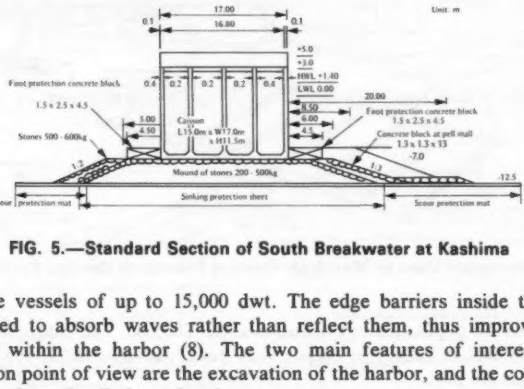

FIG. 5.—Standard Section of South Breakwater at Kashima

will handle vessels of up to 15,000 dwt. The edge barriers inside the harbor are designed to absorb waves rather than reflect them, thus improving wave conditions within the harbor (8). The two main features of interest from a construction point of view are the excavation of the harbor, and the construction of the 5-km-long (3-mile-long) breakwater.

Harbor Excavation.—Construction of the waterways involved approximately 120,000,000 cu m (157,000,000 cu yd) of excavation, mostly by suction dredging in fine sands, but also in some gravel deposits. Since offshore dumping was prohibited, all the material was pumped to land fills. The most difficult parts were the deeper waters in the outer channel. For transport to the south shore area, dredged soil was stockpiled where it was later picked up by bucket-wheel excavators and moved by a system of belt conveyors. 105,000,000 m^3 (137,000,000 cu yd) were moved in this way in a period of 3 yr. To prevent noise and dust pollution, no work was done at night, and acoustic fiberglass panels and dust covers were placed throughout the length of the conveyor system.

Construction of Breakwater.—There was no natural harbor at Kashima, so construction of the breakwater was essential for this project. Fig. 5 shows a cross section of this breakwater, which is similar to those used in other parts of Japan. It is designed for wave heights up to 7.3 m (24 ft) with a

12-sec period, and a tide fluctuation of 1.4 m (4.6 ft) in a water depth of 21 m (69 ft). The crown elevation is 5 m (16 ft) above low-water level. The base of the structure starts with a canvas sheet to minimize settlement of rubble-rock into the bed. To prevent scouring, a vinyl mat was also used at the foot. The dumped stone base is 5.5 m (18 ft) thick. The main wall is constructed of concrete caissons, most of which were prefabricated in a dry-dock [6 to 8 caisson sections, each 17 m (56 ft) wide × 15 m (49 ft) long × 11.5 m (38 ft) high on a 45-day cycle]. The caissons were floated to the site, positioned, sunk into place, and immediately filled with sand. A vinyl mat was then placed on top of the sand, and 0.7 m (2.3 ft) of concrete were placed on this from a floating concrete plant to close off the caisson. Finally, an additional 2-m (6.6-ft) thick slab of concrete was formed and placed in two layers on top of the caisson. A total of 238 caissons were placed in the south breakwater alone.

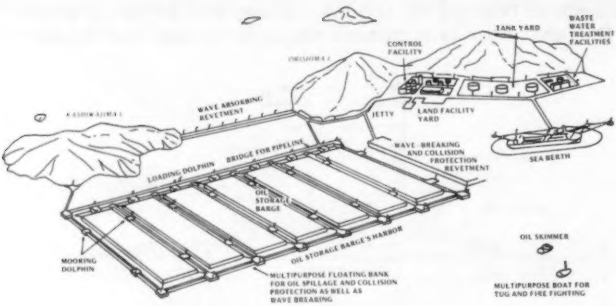

FIG. 6.—Conceptual View of Mitsubishi Floating Petroleum Storage System in Goto Islands

At present the population of the area is about 600,000. The industries generate about seven billion dollars in output each year, and 61,000,000 tons of cargo pass through the port.

Floating Petroleum Storage System

The floating petroleum storage system was proposed in response to the Ministry of International Trade and Industry's (MITI) post-1973 "oil shock" Oil Storage Law. To help stabilize supplies and prices, Japan is trying to increase domestic storage to 10,000,000 kl (63,000,000 bbl) by 1983, and to 20,000,000 kl (126,000,000 bbl) by 1986. The concept of floating concrete storage barges was developed by a venture of Mitsubishi Heavy Industries (MHI) and Kajima Corporation, and is designed to provide greater safety and have less environmental impact than surface or excavated onshore tanks. Advantages include isolation from earthquake shocks, elimination of problems resulting from foundation settlement, minimum demand for scarce land area, construction time of 2 yr versus 3 yr for equivalent land facilities, and costs comparable to land tanks. The system proposed for the Goto Islands west of Nagasaki is on a massive scale, and

would accommodate nearly one-third of the 1986 storage goals (6).

Design Concept.—The major components of the system, shown in Fig. 6, include: (1) Seven oil storage barges, each with a capacity of 835,000 kl (5,252,000 bbl), for a combined total 5,845,000 kl (36,765,000 bbl), plus barge mooring dolphins, floating oil fences, multipurpose floating banks designed for oil-spillage and collision protection as well as wave-breaking and revetments; (2) a land facility group, comprising an oil-vapor-recovery system, inert gas generating system, wastewater treatment system, and centralized control system; (3) a sea berth for loading and unloading tankers of up to 270,000 dwt class; and

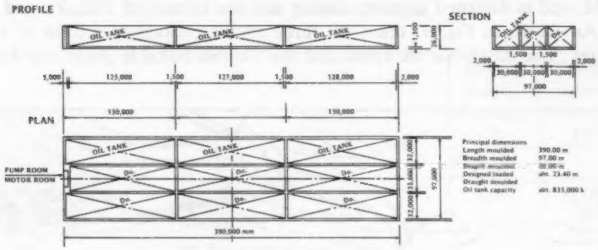

FIG. 7.—Plan and Section of Typical Oil Storage Barge (one of seven)

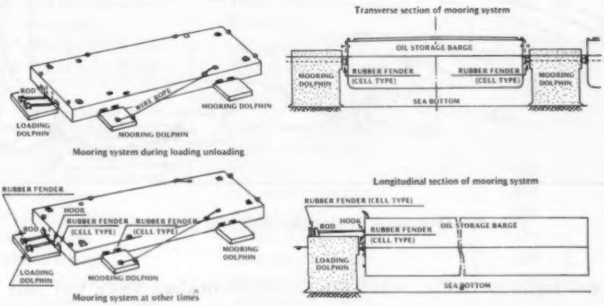

FIG. 8.—Mooring System of Oil Storage Barge

(4) small craft including tugboats, fire boats, and an oil skimmer. Design criteria include instantaneous wind velocities of up to 70 m/s (157 mph) (note that this coast experiences frequent typhoons); significant wave heights of up to 7.6 m (25 ft) with a significant period of 12 sec; and seismic considerations (6).

Barge Design and Mooring.—The dimensions for a single barge are shown in Fig. 7, and the mooring system is outlined in Fig. 8. Note that each barge is comparable in scale to the largest supertankers. A double-hull structure will be used for bottom and side panels, and the void will be filled with seawater ballast, whose greater specific gravity would prevent or minimize oil leakage

and the spread of fire in case of accidents. Each barge has nine tanks, and inert gas from the shore-based gas generator fills the space between the oil level and the rupture hatches on the decks. Fire-retarding equipment, oil-vapor-accumulation-and-reliquifaction equipment, and pump systems are also included, but are not shown on Fig. 7 (6).

Honshu-Shikoku Bridges

Three of the American engineers visited current projects on the Honshu-Shikoku Bridge System. This is one of the most ambitious construction schemes in the world, and is designed to more closely link the islands of Shikoku and Honshu (5). As shown in Fig. 9, there are three routes containing a total of 18 major bridges. The Kobe-Naruto route and the Kojima-Sakaide route will have both

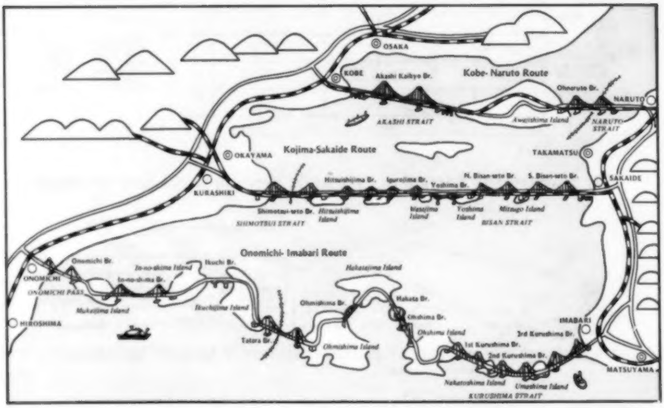

FIG. 9.—Overview of Honshu-Shikoku Bridge Project

trains and highway traffic, while the Onomichi-Imabari route will only have highway traffic. When completed, nine of these will be among the twenty longest in the world, including the world's longest, the 3,650-m (11,680-ft) Akashi Kaikyo suspension bridge, whose center span will be 1,780 m (5,840 ft). Table 1 lists the bridge structures.

Planning and field surveys for the crossings had been carried out by several agencies since 1955, but momentum really began to gather with the establishment of the Honshu-Shikoku Bridge Authority in July 1970. Operational plans for construction were approved by the Ministers of Construction and Transportation in October 1973. Construction began with the Ohmishima Bridge in December 1975, the Ohnaruto Bridge in July 1976, and the In-no-shima Bridge in January 1977. The first was scheduled for completion in 1979, and the other two by the early 1980's. Construction of the whole 18-bridge system, however, is scheduled to continue through the last 20 yr of this century.

Any project of this scale will have a major economic, social and environmental impact, and this one is especially complicated because it is entirely within the Seto Inland Sea. This is an exceptionally beautiful national park, dotted with hundreds of small, tree-covered, rocky islands, some of whose inhabitants preserve centuries-old traditions and values. It is also the major avenue of sea-borne commerce, and supports a huge fishing industry backed by a powerful fishermen's organization. The planners have tried to accommodate these and many other interests, but inevitably there have been protests and delays requiring additional revisions to the plan. The advantages for Japan's social and economic development are believed to greatly outweigh the problems, however, and all three routes are still proceeding, including the Kojima-Sakaide Route, which received considerable opposition at first.

TABLE 1.—Honshu-Shikoku Bridges

Name of bridge (1)	Type (2)	Bridge length, in meters (3)	Center span, in meters (4)
	(*a*) Kobe-Naruto Route		
Akashi Kaikyo Bridge	Suspension bridge	3,560	1,780
Ohnaruto Bridge	Suspension bridge	1,629	876
	(*b*) Kojima-Sakaide Route		
Shimotsui-seto Bridge	Suspension bridge	1,400	940
Hitsuishijima Bridge	Truss bridge	790	400
Igurojima Bridge	Truss bridge	790	400
Yoshima Bridge	Truss bridge	512	204
North Bisan-seto Bridge	Suspension bridge	1,538	990
South Bisan-seto Bridge	Suspension bridge	1,648	1,100
	(*c*) Onomichi-Imabari Route		
Onomichi Bridge	Cable stayed girder bridge	381	210
In-no-shima Bridge	Suspension bridge	1,270	770
Ikuchi Bridge	PC bridge	905	250
Tatara Bridge	Suspension bridge	1,490	890
Ohmishima Bridge	Arch bridge	328	297
Hakata Bridge	Girder bridge	335	100
Ohshima Bridge	Suspension bridge	830	550
1st Kurushima Bridge	Suspension bridge	1,224	860
2nd Kurushima Bridge	Suspension bridge	770	550
3rd Kurushima Bridge	Suspension bridge	1,520	1,000

General design criteria had to consider both typhoons [150-yr recurrent-interval wind velocities of 37 m/s to 50 m/s (83 mph–112 mph)] and earthquakes of up to 8.5 on the Richter scale. Furthermore, strong tidal-current induced forces are generated by relatively large volumes of water pouring through the constricted inlets of the Inland Sea, and currents of up to 10 knots produce whirlpools and necessitate special attention to prevent erosion of the pier foundations. Extensive model studies were undertaken, including large-scale wind-tunnel testing to help determine the structure, cross section, and configuration of each

suspension structure, and both computer and shake-table simulations of the seismic response of the structures. An additional complication was that this will be the first time in the world that high-speed trains (Japan's Shinkansen) will run on long suspension bridges, so both theoretical and model studies are aimed at anticipating structural dynamics and metal fatigue problems under these conditions, including full-scale tests with trains running on the Sanyo line.

Owing to swift tidal currents, storm-driven waves, and water depths up to 45 m (150 ft) where piers are to be located, and some complex geotechnical conditions, foundation construction is expected to be the most difficult structural task on several of the bridges. In addition to conventional caisson and cellular cofferdam methods on more favorable pier locations, the builders are employing the multicolumn foundation method and the laying-down caisson method on more difficult sites.

Construction on the Kojima-Sakaide route was delayed until 1978, however, because of the fishermen's opposition to the underwater blasting needed to excavate the foundation. Their concerns were for the environmental damage that might be caused to the fishing grounds, and the government held up construction on the Bisan-Seto Bridge pending resolution of this controversy. Even by strict American standards this concern seemed excessive to most of the visitors to the site, but it does illustrate the seriousness with which the Japanese have treated such environmental issues.

Conclusion

Particularly in coastal engineering, Americans can learn from the Japanese and could benefit through further development of cooperative relationships. Like the United States, Japan is an economically and technologically complex advanced industrial nation, and it too is seeking to accommodate social and environmental considerations in its engineering and construction planning. With its large and well-equipped research laboratories, it is able to advance systematically its technology and productivity to meet the changing needs of the domestic market.

Both private and public works projects are being planned and carried out on a grand scale and with a national determination reminiscent of the 1930's dams and bridges, the 1960's Interstate Highway System, and the manned space program in the United States. Three projects—the floating petroleum barges, the Kashima port-industrial complex, and the Honshu-Shikoku bridge system—illustrate varied aspects of planning, engineering, and construction in Japan.

Exchange programs like this one sponsored by ASCE and JSPS are an effective and economical way to keep countries informed of one another's activities. Access to research findings, design concepts, and construction procedures that prove useful in one country can help minimize wasteful duplication of efforts and help solve problems in the home countries of engineers who will take the time to study and appreciate what others are doing.

Acknowledgment

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like to thank Arthur R. Anderson, Ben C. Gerwick, Jr., John H. Nath, and John R. Paulling for their gracious hospitality and willing cooperation in interviews and in providing supplementary technical data. The writer, however, accepts responsibility for errors and opinions contained in this paper, and wishes to emphasize that the paper does not necessarily reflect official policy or opinions of the NSF or ASCE.

Appendix I.—Participants in U.S.-Japan Exchange

Dr. Arthur R. Anderson, President, ABAM Engineers, Inc., Tacoma, Washington; Speciality: Concrete structures; Visited Japan from March 15, 1978 to March 30, 1978.

Dr. Yukio Arita, Deputy Head, Hiroshima Shipyard and Engine Works, Mitsubishi Heavy Industries Co., Ltd., Hiroshima, Japan; Specialty: Tools and methods for working underwater; Visited United States from Nov. 26, 1978 to Dec. 12, 1978.

Dr. Toshio Asama, Director, Honshu-Shikoku Bridge Authority, Tokyo, Japan; Speciality: Equipment and materials for underwater construction; Visited United States from Sept. 18, 1978 to Oct. 25, 1978.

Prof. Ben C. Gerwick, Jr., Dept. of Civil Engineering, University of California, Berkeley, California; Specialty: Concrete structures for marine applications; Visited Japan from Sept. 5, 1978 to Sept. 28, 1978.

Dr. Akio Nakase, Professor of Civil Engineering, Tokyo Institute of Technology, Tokyo, Japan: Specialty: Sea bottom soils; Visited United States from Oct. 1, 1978 to Nov. 13, 1978.

Dr. John H. Nath, Director, Environmental Fluid Dynamics Laboratory, Dept. of Civil Engineering, Oregon State University, Corvallis, Oregon; Specialty: Fluid-structure interaction; Visited Japan from Jan. 31, 1978 to April 28, 1978.

Dr. John R. Paulling, Dept. of Naval Architecture, University of California, Berkeley, California; Speciality: Steel structures for marine engineering; Visited Japan from Sept. 9, 1978 to Oct. 1, 1978.

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DISCUSSION

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Discussions have a specific format. The title of the original paper/technical note appears at the top of the first page with a superscript that corresponds to a footnote indicating the month, year, author(s), and number of the original paper/technical note. The discusser's full name should be indicated below the title (see Discussions herein as an example) together with his ASCE membership grade (if applicable).

The discusser's title, company affiliation, and business address should appear on the first page of the manuscript, along with the *Proceedings* paper number of the original paper/technical note, the date and name of the *Journal* in which it appeared, and the original author's name.

Note that the discusser's identification footnote should follow consecutively from the original paper/technical note. If the paper/technical note under discussion contained footnote numbers 1 and 2, the first Discussion would begin with footnote 3, and subsequent Discussions would continue in sequence.

Figures supplied by the discusser should be designated by letters, starting with A. This also applies separately to tables and references. In referring to a figure, table, or reference that appeared in the original paper/technical note use the same number used in the original.

It is suggested that potential discussers request a copy of the *ASCE Authors' Guide to the Publications of ASCE* for more detailed information on preparation and submission of manuscripts.

Design Criteria for Floating Tire Breakwaters[a]

Closure by Volker W. Harms,[4] M. ASCE

The writer would like to thank Arunachalam and Raman for suggesting that the semi-empirical wave transmission relationship (Eq. 4) be extended to include the influence of relative draft, D/d, and also for sharing their test results for a perforated rigid floating breakwater.

It is certainly true that the development of the wave-transmission expression (Eq. 4) is based upon deep water wave conditions and that the influence of water depth, D/d, is therefore not accounted for. In view of the complexity of the dominant energy-dissipation mechanism (wave breaking, turbulent dissipation), that could not be adequately described analytically, and considering the ultimate practical value of the result, it was considered appropriate to keep the derivation relatively simple. Additionally, it was realized that sufficient experimental data on the importance of D/d, over a broad range, did not yet exist. Even now, it is doubtful that the data base is sufficient for this purpose. For these reasons the wave-transmission relationship (Eq. 4) was limited to the simplest case, deep water wave conditions. For design purposes, the writer at the present time recommends that, instead of Eq. 4, only empirical design curves such as those shown in Fig. 13 and 27 be used.

With regards to the experimental program and presentation of results, it should be recalled that the wave-height transmission ratio, C_t, depends primarily upon four nondimensional parameters (as indicated on page 159 of the paper):

$$C_t = f\left(\frac{L}{B}, \frac{H}{L}, \frac{D}{d}, \frac{B}{D}\right) \quad \text{(9)}$$

This relationship served as framework for the experimental program. For a particular breakwater, i.e., fixed B/D, this reduces to

$$C_t = f\left(\frac{L}{B}, \frac{H}{L}, \frac{D}{d}\right) \quad \text{(10)}$$

or, equivalently, since $L/d = (L/B)(B/D)(D/d)$

$$C_t = f\left(\frac{d}{L}, \frac{H}{L}, \frac{D}{d}\right) \quad \text{(11)}$$

It should be noted that d/L and L/B are not independent parameters here; only one of these may be used at a time, as in Eqs. 10 and 11. The discussers are probably quite aware of this dependence, but the discussion gives the

[a] May, 1979, by Volker W. Harms (Proc. Paper 14570).

[4] Staff Scientist, Marine Sci. Group, Lawrence Berkeley Lab., B-77-F Trailers, 1 Cyclotron Rd., Univ. of California, Berkeley, Calif. 94720.

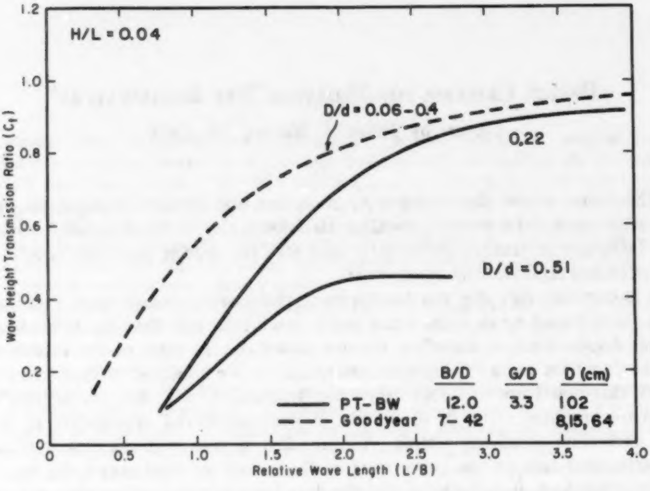

FIG. 27.—Wave-Transmission Design Curves for Goodyear and PT-Breakwaters

FIG. 28.—Assembly of PT-Breakwaters Near Large Wave Tank at CERC

impression that d/L is a further parameter to be considered in addition to L/B, which it is not.

The discussers point out, quite correctly, that the relative draft, D/d, may be of importance, particulary in shallow water. In their data for the perforated floating breakwater, Fig. 25, the influence of D/d is indeed discernible, although weak; at $H/L = 0.04$, an increase of D/d from 0.250–0.375 causes C_t to decrease

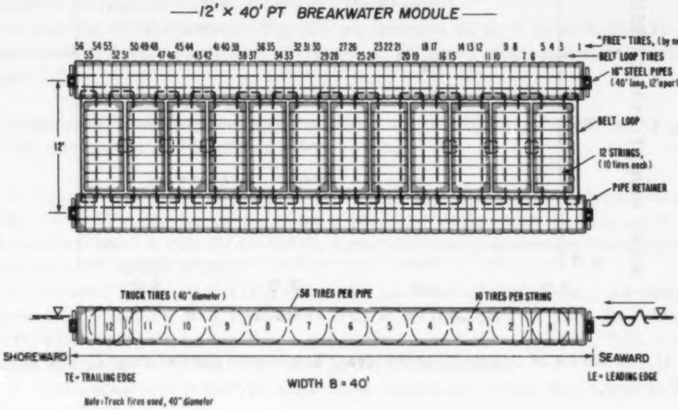

FIG. 29.—Module of PT-Breakwater (PT-1 Module)

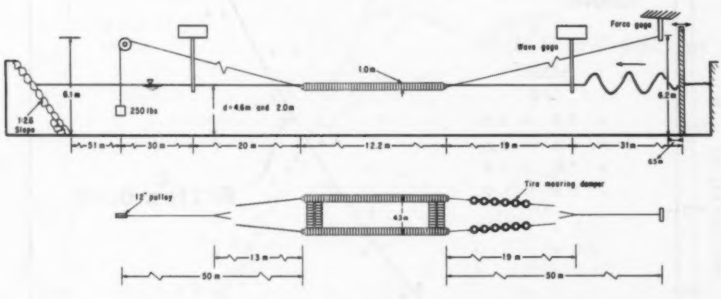

FIG. 30.—Mooring System and Wave-Tank Schematic

by less than 10%. Nevertheless, the draft parameter D/d must certainly be considered, and becomes increasingly important not only as the water depth decreases, but apparently also as the longitudinal rigidity of the breakwater increases (i.e., in the direction of wave motion). This may be demonstrated by considering the two basic types of floating tire breakwaters covered in the original paper: the Goodyear Breakwater shown in Fig. 6, which is extremely flexible along all axes, and the Pipe-Tire Breakwater (PT-Breakwater) which

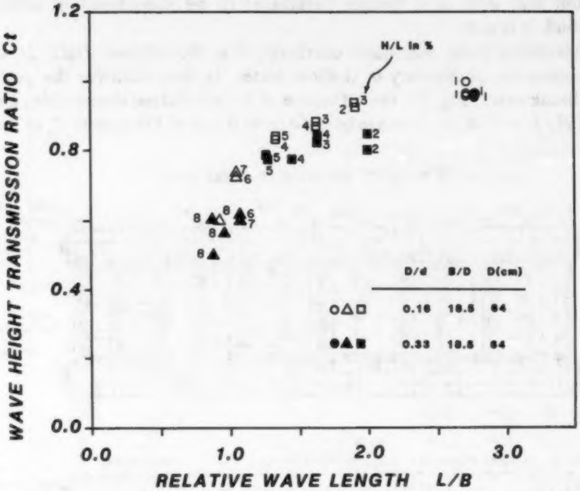

FIG. 31.—Influence of D/d on C_t of Goodyear Breakwater (for Constant L/B, H/L, B/D)

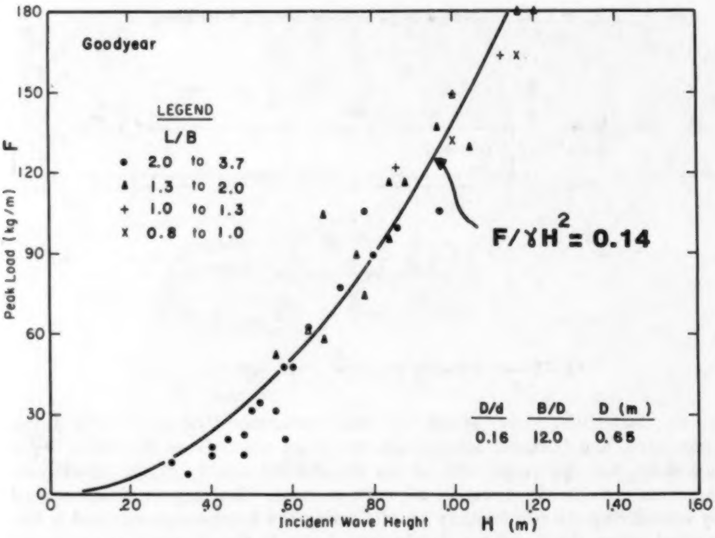

FIG. 32.—Peak-Force Data for Goodyear Breakwater

evolved out of the Wave-Guard Breakwater and, as shown in Figs. 28–30, has massive longitudinal members increasing stiffness in that direction. Considering first only prototype-scale experiments performed at the United States Army Corps of Engineers Coastal Engineering Research Center (CERC, Fig. 30), we note:

1. For the Goodyear Breakwater, Fig. 31, an increase of D/d from 0.16–0.33 causes a decrease of C_t that is typically less than 0.1.
2. For the PT-Breakwater, Fig. 27, an increase of D/d from 0.22–0.51 is associated with very substantial decreases of C_t, by as much as 0.4, depending upon L/B.

Combining this prototype-scale information with existing small-scale data (Figs. 9–11), one arrives at the following conclusions:

1. For the Goodyear Breakwater, a single wave transmission design curve (Fig. 13 or 27) may be used for most practical applications as long as D/d does not exceed 0.4; near $D/d = 0.4$ the curve will be somewhat more conservative than at lower values of D/d.
2. For the PT-Breakwater (Fig. 29), and most practical applications, the design curve corresponding to a relative draft of 0.22 may also be used for values of D/d less than this, i.e., in deeper water. For D/d in the range from 0.22–0.51, linear interpolation between curves (Fig. 27) is permissible.
3. Small-scale and prototype-scale wave transmission data was found to be in good agreement.

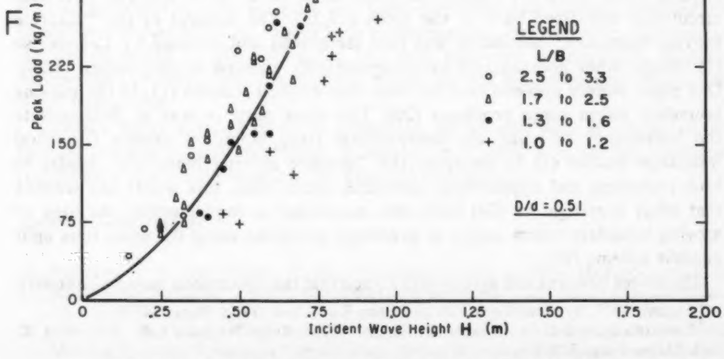

FIG. 33.—Peak-Force Data for PT-Breakwater (PT-1 Module)

Prototype-scale experiments of the Goodyear (2) and PT-Breakwaters (in preparation) indicate that the force expression given in the original paper (Eq. 7) can be simplified. Although the peak mooring force depends upon the particular type and flexibility of the mooring system, the following functional relationships between force and wave height emerged (with L/B only of secondary importance):

1. For the Goodyear Breakwater $F/\gamma H^2 = 0.14$ for $D/d = 0.16$; and $= 0.11$ for $D/d = 0.33$.
2. For the PT-Breakwater (Fig. 29 with mooring system of Fig. 30) $F/\gamma\sqrt{H^3 D} = 0.28$ for $D/d = 0.22$; and $= 0.46$ for $D/d = 0.51$.

Here F is the peak-mooring-force per unit length of breakwater and was obtained from best fit curves such as shown in Figs. 32 and 33. It is suggested that the above expressions be used as a basis for preliminary design, with values of F increased by 100 kg/m (69 lb/ft) to account for the scatter of data.

Moving Boundary Numerical Surge Model[a]

Closure by Gour-Tsyh Yeh,[4] M. ASCE and Fang-Kuo Chou[5]

The writer wishes to thank Lynch for his interest in the paper and his thorough discussion. The writer agrees with Lynch's description of the model as a "sequence of fixed-boundary simulations." Perhaps, a better term, "discrete moving boundary," may be used since this would more adequately describe a class of moving boundary models (7,20,23). The adoption of the time split explicit finite difference numerical scheme to storm surge analysis and tidal circulation has dated back to the 1960s (13,23). The concept of the "discrete moving boundary" simulation was first mentioned and outlined by Leendertse (7) though other investigators have unjustifiably claimed its proposition (1,24). Our paper simply implemented the time split explicit scheme (13) to the moving boundary storm surge problems (20). The main purpose was to demonstrate the inadequacy of using the conventional fixed boundary models for flood insurance studies (1) by applying the "discrete moving boundary" model to both prototype and hypothetical problems. Since then, this writer has learned that other investigators (24) have also succeeded in implementing the idea of moving boundary storm surges to prototype problems using the same time split explicit scheme (13).

The writer believes and agrees with Lynch that the continuous moving boundary

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[a]August, 1979, by Gour-Tsyh Yeh and Fang-Kuo Chou (Proc. Paper 14758).

[4]Research Scientist, Environmental Sci. Div., Oak Ridge National Lab., P.O. Box X, Oak Ridge, Tenn. 37830.

[5]Sr. Hydr.-Environmental Engr., Stone and Webster Engrg. Corp., Boston, Mass.

simulation as described by him would be a great improvement over the discrete moving boundary analysis if the computational algorithm could be successfully implemented. Finite element is certainly one of the techniques to tackle the problem. Other numerical methods, such as irregular grid finite difference (25) and integrated compartment method (28), that can deal with the discretization of various sizes and shapes, may also be used. It is interesting to know some investigators (27) have already initiated the employment of finite element method to handle the continuous moving boundary storm surge modeling. However, the writer is suspicious as to how long it would take to complete the implementation of the algorithm. The writer could envision several difficulties that may be encountered. One of these is that the continuous movement of the node may distort the element to the extent to cause numerical instabilities as the inundation of storm surges usually takes place in tens of kilometers. For example, Fig. 12 shows one such possibility. Initially, the element is ABCD.

After many time step computations, the element may be distorted to ABC′D′. Our experience in using the finite element coastal circulation and ground-water flow models (26,29,30) has indicated that numerical instabilities arise when the element sizes and shapes are not properly distributed throughout the region

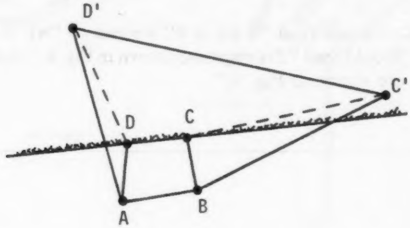

FIG. 12.—Distortion of Element

of interest (29,30,31). This fact is not surprising since the instabilities associated with irregular grid systems are quite like those associated with variable coefficients on uniform grid system (25). Under such circumstances, one may have to redesign the discretization of the region for the next time-step computation. Unless automation for discretizing the region is made, the task would become enormous if not impossible.

Lynch's paper (21) ought to be consulted by anyone who is interested in moving boundary value problems. It would be a great help to the intended researchers and should facilitate the problem formulations.

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Errata.—The following corrections should be made to the original paper:

Page 250, Eq. 8*b*: Should read "$\mathbf{n}\ \Delta\eta = 0$" instead of "$\mathbf{n}\mathbf{V} = 0$"
Page 258, line 4: Should read "Freeport are shown in Fig. 6." instead of "Freeport and Galvenston are shown in Fig. 6."

Inlet Stability Solutions for Tributary Inflow[a]

Discussion by Ashish J. Mehta,[4] A. M. ASCE and Per Bruun,[5] F. ASCE

Eq. 23 with tributary inflow $q = 0$, i.e., the parameter $e = 0.8488$, has been reported previously, using somewhat different analytical arguments (12). The inclusion of fresh water outflow in Eq. 23 is an important contribution for which the authors deserve to be congratulated. Stability evaluation for estuarine entrances such as Chandipur, Kalingapatam, Krishnapatam, Machilipatam, Nizampatam, Ponnai, and others on the peninsular coastline of India require considerations of fresh water outflow as well as wave climate and the associated littoral drift (10). These factors show a high degree of variability inasmuch as the climatic conditions during the 4 months of monsoon are markedly different

[a]November, 1979, by Francis F. Escoffier and Todd L. Walton, Jr. (Proc. Paper 14964).

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[5]Consultant, National Inst. of Oceanography, Dona Paula, Goa 403 004, India; also Consultant, International Marine Sci. and Engrg., Hilton Head Island, S.C.

from what occurs in fair weather. The monsoon climate is characterized by comparatively intense wave activity, large rates of littoral drift, and high river runoff.

An interesting case in point pertains to navigation through the Mandovi River, Goa on the West Coast of India, as shown in Fig. 8. The river entrance occurs within Aguada Bay, which is formed between Aguada and Cabo headlands. The bay enters Arabian Sea where the waves typically are on the order of 0.4 m during fair weather (18). During the southwest monsoon the waves, which are much higher, approach the river entrance by shoaling, as well by refraction and diffraction, around Cabc headland and break near the entrance where two sandy spit-like bars are formed. The breaking wave heights are on the order

FIG. 8.—Entrance to Mandovi River in Aguada Bay

of 2 m–3 m. The southern Aguada Bar is separated from the northern Reis Magos Bar by a navigation channel with a minimum center line depth of 6 m below msl. The Reis Magos Bar is partially stabilized by underwater rock reefs, but sediment movement over the two bars is believed to be strong, particularly during monsoon. Portions of both the bars are visible at low tide in fair weather (spring tide range is 2.2 m), but never in monsoon (18).

The fresh water outflow is negligible during November–May, (thus $q = 0$ may be assumed). The monthly averages for June, July, August, September and October are 40 m^3/s, 340 m^3/s, 500 m^3/s, 160 m^3/s and 60 m^3/s, respectively. The following parameters may be selected: $A = 6.01 \times 10^7$ m^2, $a_o = 1.1$ m, $a_{ci} = 3,190$ m, $L = 1,400$ m, $R_i = 4.5$ m, $T = 4.46 \times 10^6$ sec, $n = 0.018$, $\gamma = 0.80$ and $m = 1$. From Eq. 12, $E_i = 1.07$ is obtained, and

for the five values of q, the corresponding K_c are 0.63, 0.61, 0.53, 0.49, 0.58 and 0.60, respectively. The values of K_c thus range from 0.63–0.49, which is a significant variation for a single entrance. Since $K_i > K_e$ in all the five states of stability, it may be concluded that the entrance channel remains open throughout the year. It has not been possible to verify this conclusion as no bathymetric data on the bar region are available for the monsoon months. It is nevertheless difficult to imagine the channel closing under such high fresh water outflows. The fact, however, is that each year, due to wave breaking activity in the bar area, the channel is declared closed for vessel traffic by the Captain of Ports from the end of May to the middle of August, indicating the obvious role played by the waves in entrance navigation considerations. Each year towards the end of monsoon, during the feast of São Lourenço on the 10th of August, the priest of a church near Aguada symbolically outs the bar with a sword. Immediately afterwards the Captain of Ports declares the channel open for navigation.

Acknowledgement.—The writers wish to thank S. Z. Qasim, Director, and B. U. Nayak, Head, Division of Ocean Engineering, National Institute of Oceanography, Dona Paula, Goa, for their encouragement in investigating the stability of Aguada Bar.

Appendix.—Reference

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Wave Force Analysis: An Alternate Procedure[a]

Discussion by C. J. Garrison[2]

The correct values of the drag and inertia coefficients for use with Morison's equation has been a topic of considerable interest and discussion for about three decades. While the author's statement that during this time period, "little insight has been gained—in the physics of unsteady flow," disregards the basic experiments of Garrison (8), Yamomoto and Nath (9) and Sarpkaya (10), it is fair to say that there is still an unacceptable degree of uncertainty in calculation of wave forces by use of Morison's equation, and, therefore, there exists a need for further research. The author is, therefore, commended for taking a rather basic approach to a problem of great practical interest.

The following comments are directed toward the dimensional analysis presented by the author. In Eq. 1 the instantaneous force on a cylindrical member is expressed as a function of the fluid properties, ρ (density), and ν (viscosity),

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[a]February, 1980, by Yuan Jen (Proc. Paper 15166).

[2]Consultant in Marine Hydrodynamics, C. J. Garrison & Associates, 3088 Hacienda Dr., Pebble Beach, Calif. 93953.

the member diameter, D, the acceleration of gravity, g, and the instantaneous fluid velocity and acceleration, u and a, respectively. The author has carried out a dimensional analysis of the variables indicated to express the force coefficients as a function of the Reynolds number, Froude number, and Iverson number.

However, the dimensional analysis and conclusions reached from this analysis are based on a premise (Eq. 1) which is not quite complete. In oscillatory motion the force on the cylinder is not only dependent on the instantaneous fluid kinematics, u and a, as indicated by the author, but also on the immediate history of the motion. For instance, the wake left by a previous cycle of the motion represents the oncoming flow for the next and the instantaneous velocity and acceleration alone does not characterize this effect. Thus, a dimensional analysis which is to apply to practical situations must address such "history effects."

For the simplified case of sinusoidal, rectilinear flow past a circular cylinder, Garrison (8) has presented a dimensional analysis showing that C_m and C_d are dependent on the Reynolds number based on the maximum velocity during the cycle (although, the root mean square value is just as valid), the displacement ratio (which is similar to Iversen's number) and the phase angle of the sinusoidal motion. The history of the fluid motion in this instance is primarily characterized by the displacement ratio (the ratio of the amplitude of the displacement of the sinusoidal fluid motion to the cylinder radius; displacement ratio = Keulegan-Carpenter number divided by π). When the displacement ratio is in the intermediate range the wake encountered on the succeeding part of the cycle is pronounced and a large "wake effect" is found in the results. However, as the displacement ratio approaches infinity the drag coefficient should return to the steady flow value. In the experimental results which were presented in the cited papers, the variation of C_m and C_d with phase angle was disregarded and the two coefficients were presented as functions of the Reynolds number and the displacement ratio (or equivalently, the Keulegan-Carpenter number) only.

Wave induced interaction of a fluid with a cylinder is still more complex than the aforementioned situation because the fluid kinematics cannot be characterized quite so simply. In the case of simple rectilinear, sinusoidal motion only the displacement amplitude and velocity amplitude is needed to characterize the motion, whereas wave induced motion is more complex. As a consequence, it may not be possible to correlate the drag and inertia coefficients with simply the Reynolds number and displacement ratio, as in the case of rectilinear, sinusoidal motion, or the Reynolds number and Iversen number based on instantaneous fluid kinematics, as suggested by the author. At any rate, the author's contention that the instantaneous kinematics are adequate to describe oscillatory flow appears much too simplistic.

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SECOND-ORDER STANDING WAVES BOUNDED BY CIRCULAR CYLINDERS[a]

Discussion by Michael de St. Q. Isaacson,[4] M. ASCE

In order to clarify the boundary condition discontinuity described in Ref. 7 pertaining to wave diffraction past a vertical circular cylinder, the authors have described a second-order Stokes solution to a related problem. As already pointed out by the writer (9,22), the inconsistency does not itself imply, as originally stated in Ref. 7, that a rigorous solution is not possible, but rather that it in fact corresponds to a discontinuity, or irregularity, in the second-order radial velocity.

Indeed the second-order forces for the case of incident wave scattering, which is the case of primary engineering interest, have been expressed by Lighthill (20) without invoking a detailed solution of the second-order flow field. The second-order velocity potential ϕ_2, itself, only influences the second-order forces through the component

$$F_2^{(a)} = \epsilon^2 \int_{S_0} \rho \frac{\partial \phi_2}{\partial t} \mathbf{n} \, dS \quad \text{(41)}$$

in which $\mathbf{n}$ = the unit vector normal to the body surface; S_0 = the surface area of the body below the plane $z = 0$; and dS represents an element of S_0. Other second-order force components are associated with the dynamic pressure distribution and also with the effect of extending integrations up to $z = \eta$ rather than $z = 0$, but both these components depend only on the first-order potential ϕ_1 and present no real difficulty.

On the basis of Green's theorem, Lighthill (20) has shown that the component $F_2^{(a)}$ may be expressed in the form

$$F_2^{(a)} = -\rho \int_{S_f} w(\nabla \phi_1)^2 \, dS_f \quad \text{(42)}$$

in which w = the vertical velocity at $z = 0$ due to the flow which corresponds to the body oscillating in an otherwise stationary fluid with unit amplitude in the direction of $F_2^{(a)}$ and at a frequency of 2ω; ϕ_1 = the first-order potential of the diffraction problem under consideration; and S_f corresponds to the plane $z = 0$ exterior to the body.

Other solutions have also been obtained, as mentioned in Ref. 23, but comparisons of alternative solutions are still needed.

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[a]February, 1980, by John N. Hunt and Rafik E. Baddour (Proc. Paper 15166).

[4]Assoc. Prof., Dept. of Civ. Engrg., The Univ. of British Columbia, Vancouver, B.C., Canada V6T 1W5.

Journal of the Waterway, Port, Coastal and Ocean Division, ASCE, Vol. 105, No. WW4, Proc. Paper 14940, Nov., 1979, pp. 447–478.
23. Isaacson, M. de St. Q., discussion of "Irregularities in Solutions of Nonlinear Wave Diffraction Problem by Vertical Cylinder," by T. Miloh, *Journal of the Waterway, Port, Coastal and Ocean Division,* ASCE, Vol. 107, No. WW2, Proc. Paper 15385, May, 1980, pp. 279–284.

C & D Canal Effect on Salinity of Delaware Estuary[a]

Errata

The following corrections should be made to the original paper:

Page 14, Fig. 11 should be replaced with:

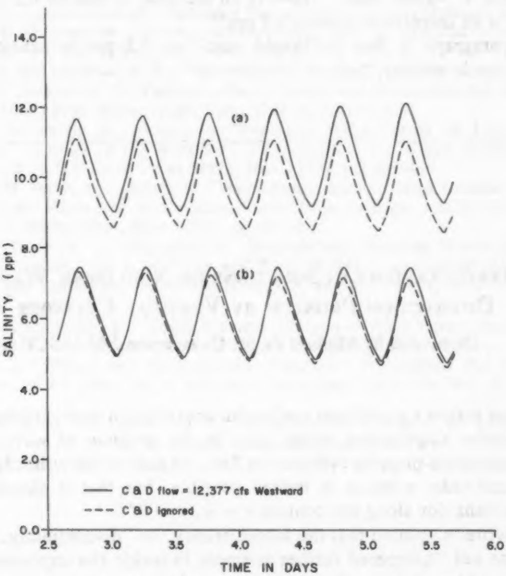

FIG. 11.—Delaware Estuary, Salinity Time Series Comparison at Two Locations: (*a*) 4.8 miles (7.7 km) Downstream of Reedy Pt.; (*b*) 6.8 miles (10.9 km) Upstream of Reedy Pt.

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[a]February, 1980, by Tavit O. Najarian, M. Llewellyn Thatcher, and Donald R. F. Harleman (Proc. Paper 15172).

Page 14, Fig. 12 should be replaced with:

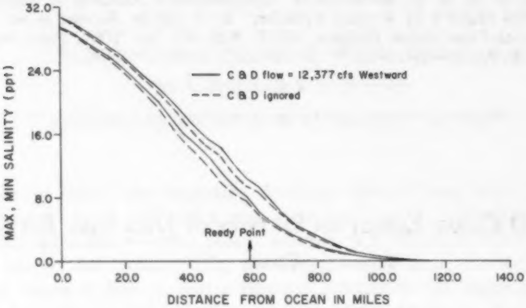

FIG. 12.—Delaware Estuary, Salinity Profile Comparison During 10th Tidal Cycle: Canal Flow Westward

Page 15, line 4: Should read "There is an increase of almost 0.2 ppt" instead of "There is an increase of almost 1.8 ppt"
Page 15, paragraph 4, line 5: Should read "of 0.2 ppt in salinity," instead of "of 1.8 ppt in salinity,"

Irregularities in Solutions of Nonlinear Wave Diffraction Problem by Vertical Cylinder[a]

Discussion by Michael de St. Q. Isaacson,[2] M. ASCE

The author makes a significant and useful contribution in describing the nature of second-order singularities which arise in the problem of waves diffracted by a vertical surface-piercing cylinder. In Refs. 14 and 15, the writer has indicated that a second-order solution is indeed possible, but that it should "account for singular behavior along the contour $r = a$, $z = 0$."

It is the writer's opinion that the inconsistency, or "discontinuity," described in Ref. 4 has not "hampered further attempts to tackle the important nonlinear diffraction problem," but rather has attempted to contribute towards a better understanding of the problem and towards the development of a completely rigorous second-order solution. Indeed, the author does suggest that the previous

[a] May, 1980, by Touvia Miloh (Proc. Paper 15385).

[2] Assoc. Prof., Dept. of Civ. Engrg., Univ. of British Columbia, Vancouver, B.C., Canada V6T 1W5.

solutions of Refs. 1, 2, 7, 8, 10 "appear to be incorrect in the sense that they do not consider the irregularity . . ." It is unfortunate that the discussions of Ref. 4 given by Garrison (12) and by Venkatanarasaiah (19) attempted to resolve the complication by decomposing ϕ_2 into forms such as $\phi_2 = \phi_2^{(i)} + \phi_2^{(s)}$ etc., in themselves, these clearly do not deal with the problem, but show that it had not been fully appreciated.

Additional solutions to the second-order problem have since been proposed by Chen (11), Hunt and Baddour (referred to in Ref. 13), Kurata and Ijima (17), Lighthill (18) and M. Raman (personal communication, 1979). The author is referred to a companion discussion, Ref. 16, which summarizes Lighthill's solution in which a complete description of the second-order flow field is not needed.

It appears that continued effort is still needed to compare alternative solutions and to arrive at some consensus concerning their validity.

Appendix.—References

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Perturbation Methods in Diffraction[a]

Discussion by Michael de St. Q. Isaacson,[2] M. ASCE

The comments made by the author are of considerable interest and value to those concerned with nonlinear wave interactions with large bodies. However, the existence of a formal second-order solution to the stated diffraction problem is not in contention. Subsequent to Ref. 1, the writer (4) has suggested that "any proposed second-order solution should account for singular behavior along the contour $r = a$, $z = 0$" and that (5) "the inconsistency (referred to) implies that an exact second-order Stokes solution should exhibit a discontinuity in second-order radial velocity . . . rather than that no solution exists." Further solutions to this second-order diffraction problem have now been obtained by several authors as mentioned in the companion discussions, Refs. 6 and 7.

Furthermore, the writer is not "proposing to judge the validity of a particular method of finding approximate solutions of a mathematical problem by comparisons (with measured values)." It is suggested, though, that a second-order solution based on the Stokes expansion procedure should be useful for engineering purposes over relatively restricted conditions, as in the case of an undisturbed wave train. For example, nonlinear effects are of paramount importance in offshore design for steep waves in shallow water, which are precisely the conditions under which second-order Stokes theory becomes invalid. In the particular case of the inertia force limit to the diffraction problem under consideration, the maximum force predicted by second-order theory is about, say, 5% greater than the linear theory prediction for typical design wave steepnesses in deep water. On the other hand, in shallow water the maximum inertia force predicted by nonlinear shallow wave theory is typically much greater than that predicted by linear wave theory, so as to make the second-order results questionable in such applications.

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[2]Assoc. Prof., Dept. of Civ. Engrg., Univ. of British Columbia, Vancouver, B.C., Canada V6T 1W5.

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INLINE FORCES ON FIXED VERTICAL CYLINDERS IN WAVES[a]

Discussion by Steven E. Ramberg[2]

The author is to be commended for a thorough presentation of his experimental results for the inline forces on vertical cylinders in regular waves. The purpose of this discussion is to seek further insight into systematic variations of drag and inertia coefficients and to examine an apparent discrepancy between the results presented by the author and some earlier data (13).

Since the author recorded eight to nine consecutive cycles of force data and then obtained C_m and C_D coefficients for each cycle, it would be useful to know of the magnitudes and possible interrelationships of cycle-to-cycle variations in the computed coefficients. If these variations are compared to an estimate of the overall measurement uncertainty, then it may be possible to reconcile reports of such variations (8,9,11) with a report of a lack of cycle-to-cycle variations in planar harmonic flow (5). On page 149 the author mentions, without explanation, that different values of u_m were employed for each cycle; was this related to the foregoing problem or to some other feature of the experiments?

In a recent paper (13), we reported that an axial distribution of planar flow results taken from Sarpkaya's data significantly over-predicted the drag component of the inline force on a vertical cylinder in regular waves. Our result conflicts with the present finding unless the force coefficients in waves are dependent on more quantities than are listed in Eq. 4. One additional effect, among several which have been discussed elsewhere, (10,12), is the combined influences of kinematical gradients along the cylinder axis and of variations in particle orbit eccentricity from one case to another. The present experiments cover a wider range of wave/cylinder conditions than our study but tend toward the shallow water regime, whereas our conditions were virtually all deepwater cases for which the kinematic gradients are much greater and the orbits more circular. While these general differences in wave conditions can qualitatively account for the differences in the findings, it is possible that a more quantitative accounting can be obtained from further examination of the author's data. Specifically, it would be useful to know the correlation between the magnitudes of the velocity gradients over the measurement cylinders and the deviations between the results measured in a wave field and the planar flow data shown in Fig. 7 of the paper.

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[a]May, 1980, by Subrata K. Chakrabarti (Proc. Paper 15403).

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